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# Nonlinear Analysis of a Building Surmounted by a RC Water Tank under Hydrodynamic Load

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***Abstract*****—**In this paper, we study a complex structure which is an apartment building surmounted by a reinforced concrete water tank. The tank located on the top floor of the building is a container with capacity of 1000 $m^3$. The building is complex in its design, its calculation and by its behavior under earthquake effect. This structure located in Algiers and aged of 53 years has been subjected to several earthquakes, but the earthquake of May 21st, 2003 with a magnitude of 6.7 on the Richter scale that struck Boumerdes region at 40 Kms East of Algiers was fatal for it. It was downgraded after an investigation study because the central core sustained serious damage. In this paper, to estimate the degree of its damages, the seismic performance of the structure will be evaluated taking into account the hydrodynamic effect, using a static equivalent nonlinear analysis called pushover.

***Keywords*****—**Performance analysis, building, reinforced concrete tank, seismic analysis, nonlinear analysis, hydrodynamic, pushover.

## I. INTRODUCTION

THE earthquake of May 21st, 2003 that hit the Boumerdes region at 40 km East of Algiers has caused considerable damages to the building surmounted by a tank which is the subject of our study (Fig. 1).The present study focuses on the assessment of the seismic performance of this complex structure. It is located in the municipality of Dar El Beida, classified as strong seismicity zone by the Algerian seismic code [1]. First, this structure has been the subject of research undertaken by Hamitouche et al. [2]. A linear analysis was conducted by modeling the structure using the finite element technique and using the structural finite element software Robot©. Thus, they were able to identify the causes of damages on the central core of the structure caused by the earthquake of Boumerdes. One of the main results highlighted in their study is that the mapping of cracks obtained by the numerical simulation when the tank was full, agree well with cracks found on site during the investigation. The study concluded that the tank was full at the moment of the earthquake.

After that, a second work has been undertaken by Ait L'Hadj et al. [3], in which the seismic performance of the structure was evaluated taking into account the hydrostatic effect, using a static equivalent nonlinear analysis called pushover exposed in the ATC40[4]. The maximum displacement of the structure and the degree of its damage were estimated. The research concluded that the studied structure is classified in the third domain which corresponds to a plastic behavior.

In order to best approximate the structure behavior, we need to evaluate its seismic performance taking into account the hydrodynamic effect. The present paper extends the work of Ait L'Hadj et al. [3]. Section II contains a general description of the structure (building-tank). Section III describes the hydrodynamic effect assessment by the method of Housner [6]. Section IV contains to the results discussion after pushover analysis. Finally, the main conclusions and the lessons learnt from this analysis are given in Section V.

## II. DESCRIPTION OF THE BUILDING - TANK

The building has a form of square tower of 17 m of side, seven floors surmounted by a tank with a container capacity of 1000 $m^3$. This construction consists of a cylindrical central tower of 6 m of diameter, supporting the tank and around which are arranged in a star eight concrete walls forming the frame. The bracing is provided by the floor. The central tower includes the stairs, elevator, levels of access to the apartments, and the ducts through which pass tank pipes. The tank is constituted of a container in the paraboloid of revolution form connected to a spherical dome. The facades do not include bricks, at least externally, but largely use glass, plastic and aluminium. The ground floor is open and can be used partially as shelter to vehicles. The construction started in October 1960; the building site was practically finished in January 1962.

The storage tank has a height of 8 m from the reservoir bottom. Its walls have a thickness varying from 25 cm at the bottom of the tank to 10 cm at the top of the tank. The cover dome has the form of a spherical cap, with a thickness of 10 cm. All the floors are made of solid slab of 12 cm in thickness; the roof floor is inaccessible and has a sealing complex system with a slope to facilitate the flow of rainwater. The central core supporting the tank is of cylindrical form, with a thickness of 15 cm. The 8 concrete walls forming the frame, arranged in a star, have 2 m in length, and a thickness of 25 cm. The beams have a high rigidity, because the dimensions are relatively large, 25x80 cm² for the beams of the ground floor, 15x60 cm² for the beams of the current floor.

## III. HYDRODYNAMIC EFFECT ASSESSMENT BY THE METHOD OF HOUSNER [6]

The evaluation of hydrodynamic forces in the tank is a crucial step. For this reason, we use the analytical method of

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Housner [6] to estimate the response of a liquid in rigid tanks, under seismic excitation. In the case of a water storage elevated tank, we cannot consider the container as being rigidly related to the soil and therefore undergoing the same acceleration as the soil. Indeed, when the container is on top of a structure, we must consider the flexibility of the latter. The approach of Housner consists of modeling the water contained in the tank by its equivalent mechanical and mathematical model as shown in Fig. 2.

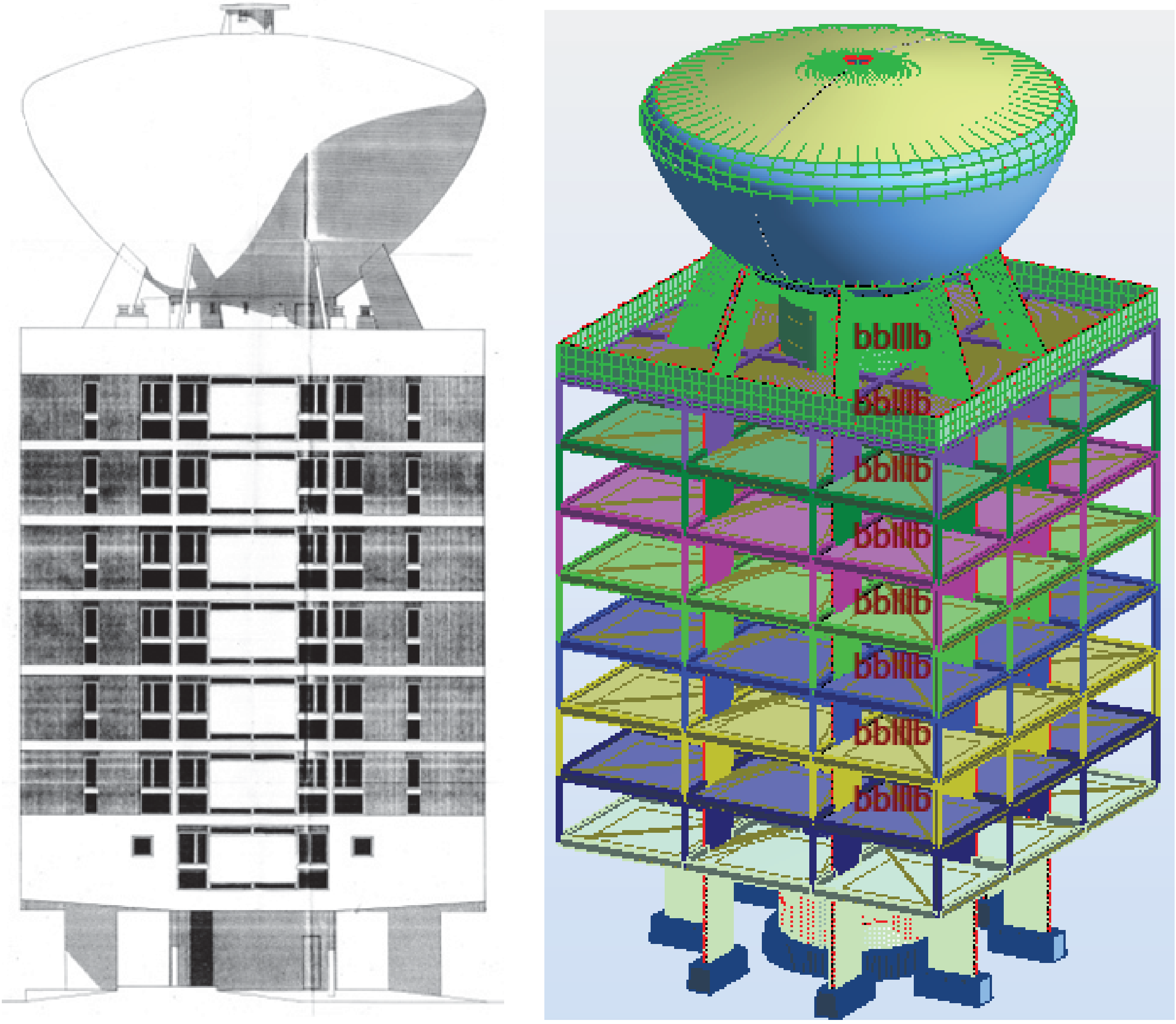

Fig. 1 General view and longitudinal cross section of the building-tank [5]

The action of the liquid is divided into two actions: A passive action causing pulse actions and active action causing oscillation actions [7]. The pulse efforts come from of that a portion of the liquid mass, called passive mass, reacts by inertia to the translation of a tank walls. The mechanical system of the container is obtained by considering one part of the liquid mass, noted $M_8$, said passive mass, rigidly connected to the container and an another part of the liquid mass, noted $M_9$, said active mass (see Fig. 2). In summary, the total water mass $M_e$ can be divided into a passive mass $M_8$ and an active mass $M_9$ rigidly connected on the one hand and by means of a spring of spring constant $K_1$ on the other hand.

In the adopted mathematical model for a building surmounted by a tank (Fig. 2), the mass $M_9$ is connected to the structure by a rod of the same stiffness $K_1$ ensuring direct coupling with $M_8$, while $M_8$ is connected to the building by a rod representing the supporting system of the last floor of spring constant $K_0$. The building-tank has nine degrees of freedom.

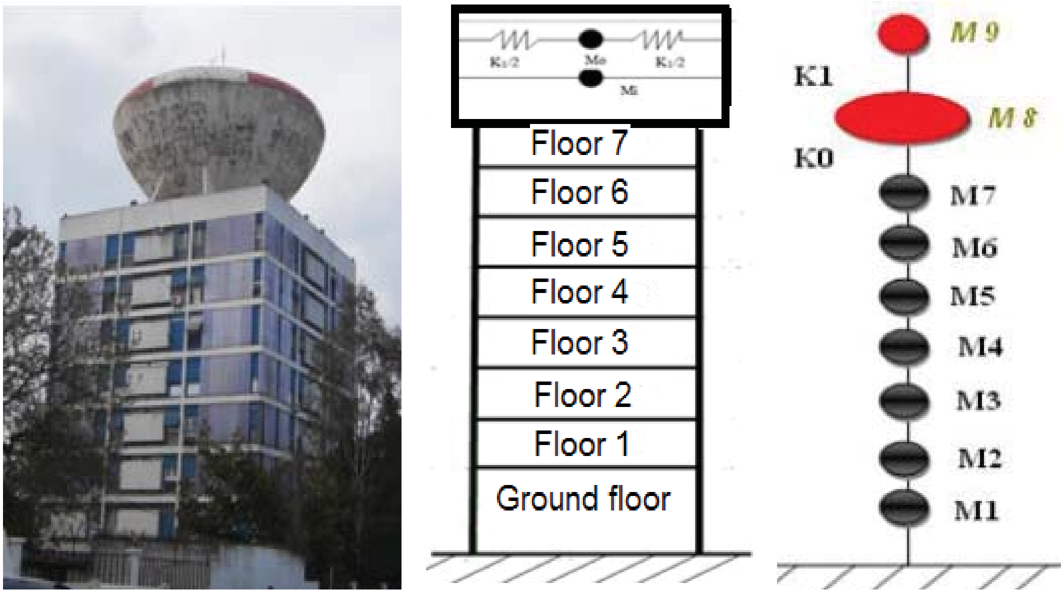

Fig. 2 Building-tank, mechanical system and mathematical model

## IV. STATIC EQUIVALENT NONLINEAR ANALYSIS: PUSHOVER ANALYSIS

An analysis is performed on the structure to determine its behavior in the nonlinear domain and assess its seismic performance. After this nonlinear analysis, we will determine the relative displacements between floors and the demand of ductility $\mu_D$ that characterizes the penetration degree of the building in plastic domain. We will also determine the distribution of plastic hinges in different structural elements as beams and columns and their levels of damage corresponding to the performance points.

### *A. Capacity Curve*

The capacity curve reflects the ability of the structure to resist to earthquake and represents the shear at the base of the structure as a function of the displacement of the latter. This curve is formed by a linear elastic phase followed by a nonlinear phase corresponding to the plastic hinges formation.

The capacity spectrum of the structure in the format ($S_a$-$S_d$) obtained with the structural finite element software Etabs© [8] is shown in Fig. 3.

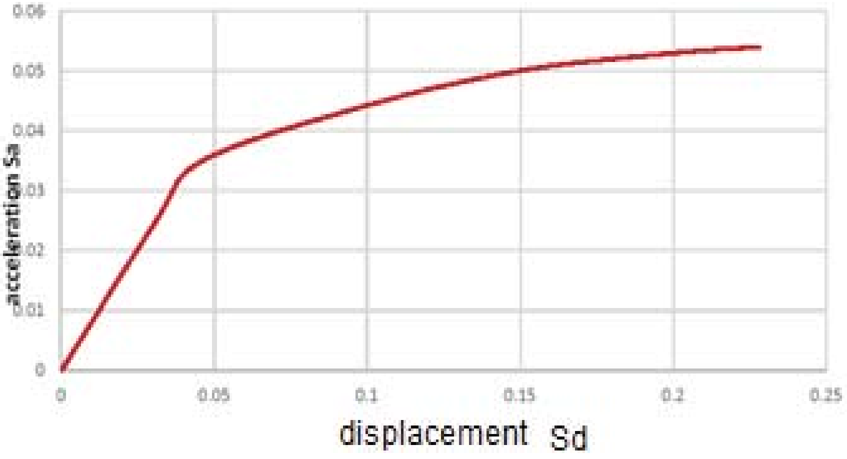

Fig. 3 Capacity spectrum of the structure (building-tank)

### *B. Plot of the Bilinear Curve*

The bilinear representation of the structure capacity curve is developed according to the procedure of the ATC40[4]. The result is shown in Fig. 4 and the coordinates of the points A and B of the bilinear curve are given in Table I.

TABLE I
THE COORDINATES OF THE POINTS A AND B

| Point | $S_d$ (m) | Observation |
|---|---|---|
| O | 0.000 | Origin point |
| A | 0.0544 | $S_{dy}$: Elastic limit of displacement |
| B | 0.2280 | $S_{di}$: Ultimate limit of displacement |

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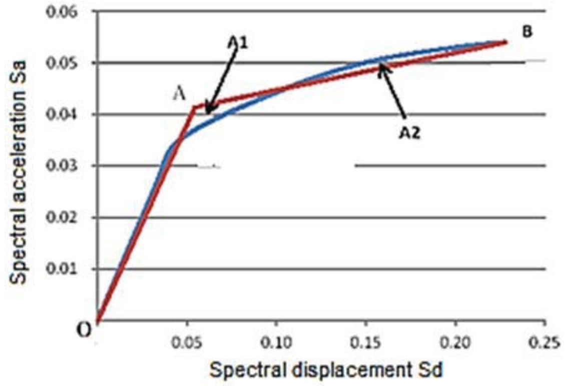

Fig. 4 Bilinear representation of the structure capacity spectrum

The effective damping $\xi_{eff}$ calculation for a series of values ($S_d$, $S_a$), according to the relations given in the Algerian seismic code, allowed us to draw the curve ($\xi_{eff}$ - $S_d$) as shown in Fig. 5.

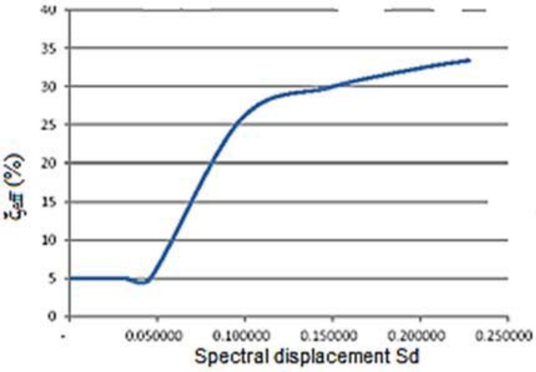

Fig. 5 Representation of the curve ($\xi_{eff}$ - $S_d$)

## C. Response Spectrum

The response spectrum as a representation of the seismic acceleration evaluation as a function of the period ($S_a$-T) is given in the Algerian seismic code, Fig. 6.

The elastic response spectrum in the traditional format, as shown in Fig. 6, is transformed in to acceleration spectrum format $S_a$ as a function of the displacement spectrum $S_d$ (Fig. 7) with viscous damping constant fixed at 5%, using the relations given in [4].

## D. Evaluation of the Performance Point

A family of reduced spectrum for $\xi_{eff} \geq 5\%$ are developed and shown in Fig. 8. By superposing the bilinear spectrum capacity curve of Fig. 4 with the reduced spectrum family, we deduce the intersection points ($\xi_{eff}$ - Sd) as represented in Fig. 8. At this stage, we superpose the two curves ($\xi_{eff}$- $S_d$). The first curve is given in Fig. 5 and the second curve is obtained from the values taken from Fig. 8. So, we deduce the performance point as shown in Fig. 9, which has as coordinates (0.09, 23.5%).

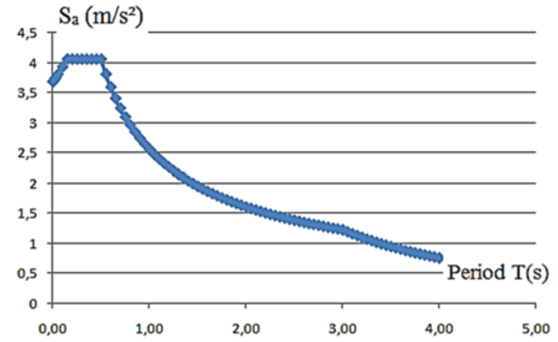

Fig. 6 Elastic response spectrum in the traditional format ($S_a$-T)

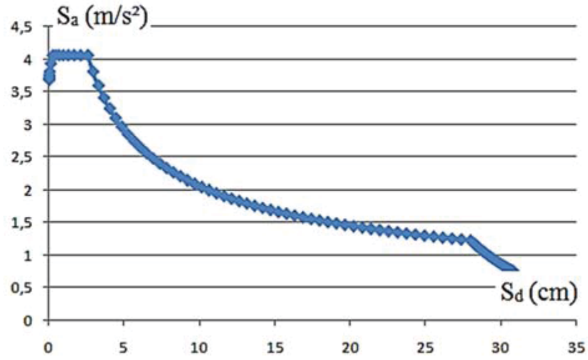

Fig. 7 Elastic response spectrum in the format ($S_a$ - $S_d$)

## E. Evaluation of the Damage Domain

The European code EMS 98[9] proposes a scale of five domains to illustrate the levels of structure damage according to Table II. Each performance point can be located in an interval which defines a state of structure damage. These five domains are shown in Fig. 9. The European macroseismic scale of EMS 98[9], has allowed us to classify our structure in the third damage domain; which describes a very important state of damage in the building structural elements. In reality, these damages are manifested by the cracks in the central core of the building at the ground floor as noticed by Hamitouche et al [2]. Even the study undertaken by Ait L'Hadj et al. [3], under hydrostatic affect, concluded that the structure is classified in the same damage domain (level 3).

TABLE II
DISPLACEMENT LEVELS OF DIFFERENT DAMAGE DOMAINS

| **Damage domains** | **Relations of the different spectrum displacement** | **$S_d$ (m)** |
|---|---|---|
| Domain 1 | $S_d$= 0,4 $S_{dy}$ | 0.0220 |
| Domain 2 | $S_d$= 0,8 $S_{dy}$ | 0.0435 |
| Domain 3 | $S_d$= $S_{dy}$+ 0,25 ($S_{di}$– $S_{dy}$) | 0.0975 |
| Domain 4 | $S_d$= 0,75 $S_{di}$ | 0.171 |
| Domain 5 | $S_d$= $S_{di}$ | 0.228 |

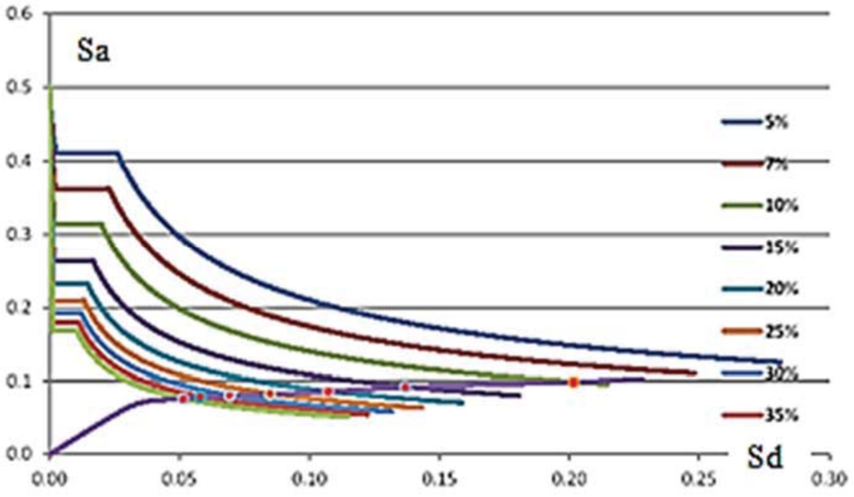

Fig. 8 Representation of the reduced spectrum family ($\xi_{eff}$ - $S_d$)

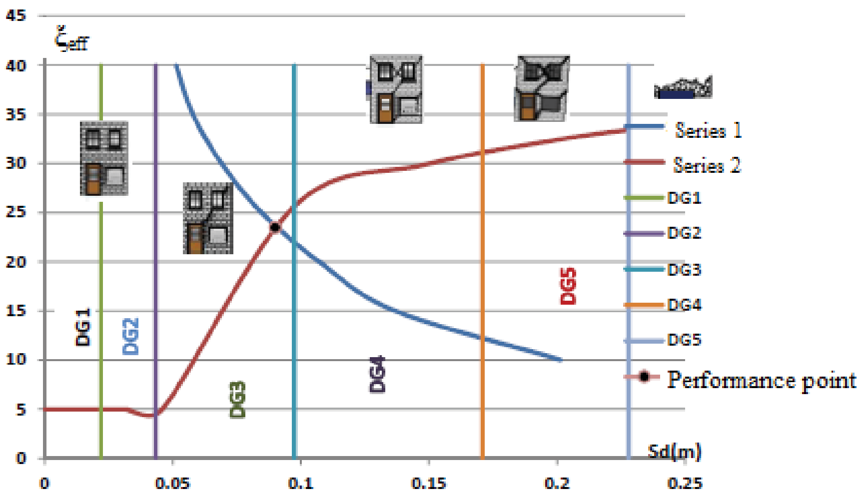

Fig. 9 Determination of the performance point

### *F. Displacements between Floors*

According to the Algerian seismic code [1], lateral relative movements of a floor with respect to the adjacent floors should not exceed 1% of the floor height. The relative displacement between the floor (k) and the floor (k-1), which is recognized as an important indicator of building performance because it is directly related to the maximum stress developed in the plastic hinges and damage in non-structural elements is given by:

$$\Delta U_k = U_k - U_{k-1} \tag{1}$$

For our building, the height of the current floor and the ground floor are respectively 2.87 m and 4.80 m. The verification of displacements between floors revealed that the condition of the Algerian seismic code is satisfied as shown in Fig. 10.

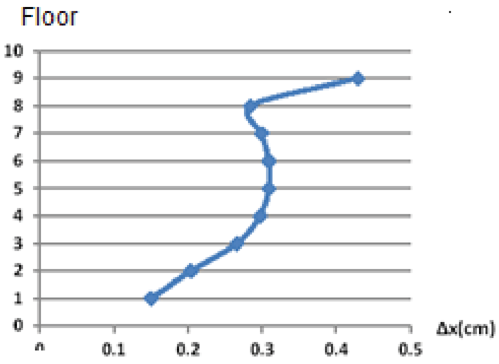

Fig. 10 Displacements between floors

### *G. Ductility Demand*

The ductility demand is a criterion which is the penetration degree of the structure in the post-elastic domain; it is estimated by the ratio between the displacement corresponding to the performance point and the elastic displacement.

The ductility, which reflects the ability of the structure to dissipate the energy in entering in plastic deformations, shown that the structure has widely penetrated in the plastic domain (see Table III). Hinges are widely formed and their plasticization is advanced. The structure studied has globally a plastic behavior. This confirms the damage domain of level 3.

TABLE III
EVALUATION OF THE STRUCTURE PENETRATION DEGREE IN THE PLASTIC DOMAIN

| | |
|---|---|
| $S_d$ displacement at the performance point | 0.090 |
| $S_{dy}$ elastic displacement | 0.0544 |
| $\mu_D$ | 1.654 |

### *H. Failure Mechanism and Degradation States of the Structure*

The structural finite element software Etabs© [8] allows us to visualize the development of the plastic hinges in the structural components. The analysis of the distribution map of plastic hinges shows the appearance of plastic hinges of type D in the third floor of the central core (Orange color in Fig. 11). This type of hinges means that the collapse step has been exceeded. The analysis under hydrostatic load undertaken by Ait L'Hadj [3] concluded that plastic hinges of type CP which means a Collapse Prevention level (green color in Fig. 11) appeared in the third floor, but the collapse step has never been reached.

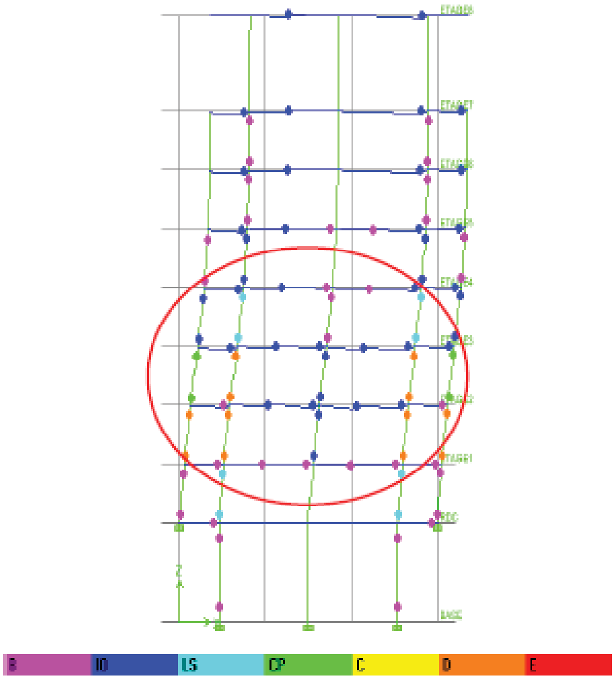

Fig. 11 Appearance of hinges of type D in the central core

## V. CONCLUSION

The nonlinear analysis demonstrates that the performance point of the structure is located in the third damage domain. The value of its spectral displacement (0.09 m) given by this study, under hydrodynamic effect, is greater than the value of spectral displacement (0.059 m) given in [3], under hydrostatic effect. We demonstrate, too, that under hydrodynamic effect, the plastic hinges have exceeded the collapse level, in contrary to the nonlinear analysis under hydrostatic load [3], which concluded that the plastic hinges have not exceeded the collapse level.

We can conclude that the nonlinear analysis under hydrostatic load underestimates the spectral displacement of the performance point, and the penetration rate of the plastic hinges in the plastic domain. So, the nonlinear analysis under hydrodynamic load approaches the real behavior of the studied structure under seismic action. Because, it takes into account the water effect in the container of the tank, which effect is not negligible under seismic effect.

We have treated in this paper a very complex structure, by the dual functionality (building for residential use and water storage tank) that it fulfils and by its behavior mode according to the influence of the hydrodynamic effect of the water contained in the tank.

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